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Geogrid Reinforced Soil Retaining Wall on Compressible Soils

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SYNOPSIS: A geogrid reinforced soil wall with a wrap-around facing system was successfully constructed on soft, compressible alluvial and residual soils. An originally designed 20-foot (6.1 m) high, reinforced concrete cantilever retaining wall was not constructed because of the expected settlements induced by the wall and backfill. The geogrid reinforced wall was utilized because of its ability to withstand the estimated settlement and because it was considered less expensive than providing deep foundation support of a cantilever wall. This paper discusses the design, construction, and performance of the geogrid reinforced wall.

INTRODUCTION

A 20-foot (6.1 m) high, reinforced concrete cantilever retaining wall was planned adjacent to a proposed clubhouse as part of the site development for a large luxury apartment complex near Atlanta, Georgia. This wall would separate the clubhouse pool area from a small lake to be constructed in a creek channel. During initial grading operations in the proposed retaining wall area, the contractor noticed that the subgrade was very soft. A geotechnical consultant was then retained by the owner to investigate the subsurface conditions in the wall area and to make recommendations concerning design and construction of the wall.

SUBSURFACE INVESTIGATION

Three soil test borings were performed along the proposed curvilinear retaining wall alignment. Standard penetration tests were conducted in the borings at intervals of 2.5 to 5.0 feet (0.8 to 1.5 m). All soil sampling and standard penetration testing were in general accordance with ASTM standard D 1586. The boring data indicated up to 27 feet (8.2 m) of generally soft or loose soils. Alluvial soils were encountered to depths of up to 8 feet (2.4 m) below the existing ground surface. The alluvial soils were deposited by the adjacent creek and typically consisted of fine sandy clay or clayey fine to medium sand (Unified Soils Classification of CL and SC). Residual soils were encountered below the alluvium to boring termination depths. Residual soils are defined as materials formed in-place by the chemical weathering of the parent rock (metamorphic rocks underlying the site include gneiss, amphibolite, and schist). The residual soils were typical of those found in the Piedmont Physiographic Province and generally consisted of micaceous fine sandy silt (ML) and silty fine sand (SM). Standard penetration resistances in the

alluvial and residual soils typically varied from 5 to 11 blows per foot in the compressible zone (upper 27 feet of soils). The ground water level was measured at 0 to 6 feet (0 to 1.8 m) below the ground surface in the borings.

Based on the subsurface data obtained and subsequent analyses, it was estimated that total settlements of up to 3 inches (7.6 cm) would occur due to the weight of the required fill behind the wall. Because of varying subsurface conditions and varying wall heights, differential settlements of up to 2 inches (5.1 cm) were estimated. Since it was also estimated that 60 to 70 percent of the total settlement would occur during fill placement, pre-loading was initially considered. However, time constraints set by the owner/developer eliminated pre-loading as a possible alternative. Because of the amount of differential settlement expected and the project time constraints, a conventional spread foundation for the cantilever retaining wall was not feasible.

Two alternatives were then considered for wall construction. First, using the original cantilever retaining wall design, a deep foundation system would be required. Timber piles were considered to be the most economically feasible deep foundation system. The second alternative was to use a flexible wall system that would tolerate the estimated settlement. A polymer geogrid reinforced soil wall with a wrap-around facing was evaluated as the flexible wall system (TENSAR, 1986). The second alternative was chosen by the owner based on economics.

PROJECT REQUIREMENTS

The required retaining wall consisted generally of two semi-circular segments with radii of 16 and 68.5 feet (4.9 and 20.9 m). The smaller radius wall was to be about 8 feet (2.4 m) high while the large radius wall height varied from 2 to 20 feet (0.6 to 6.1 m). Figure 1 shows a plan view of the proposed walls. Since a vertical wall was only required above the proposed lake level, the maximum wall height was changed to 10 feet (3.0 m) supported by a reinforced slope up to 10 feet (3.0 m) high. For architectural reasons, a cast-in-place concrete facing was required for the wall. The concrete facing was designed as a free-standing member subjected to no earth pressure from the geogrid reinforced soil wall. Following construction of the flexible wall and settlement monitoring, the cast-in-place concrete facing would be constructed.

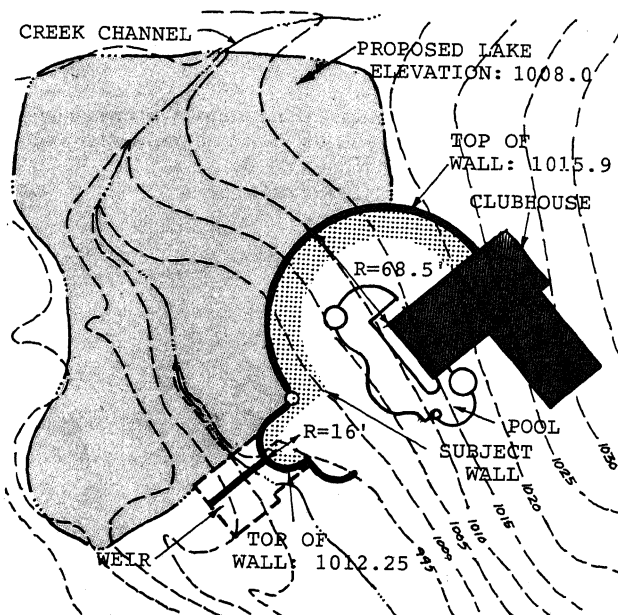


Figure 1. Plan View of Wall

DESIGN

The geogrid reinforced wall was designed using the tie-back wedge method of analysis. It is assumed that active lateral earth pressures are developed for polymer reinforced walls (Jones, 1985). These pressures are then resisted by the tensile force of the reinforcement. The kinematic mechanism of the wall is rotation about a hinged toe and pressures from the backfill retained behind the reinforced mass are also considered in the analysis (Berg, et al, 1986). In addition to internal stability, external stability modes of sliding, overturning, and toe bearing failures were checked using retaining wall analysis techniques.

Overall or global stability of the retaining wall and underlying slope was evaluated using the "Newslope" computer program. This program

considers circular failure surfaces and uses the modified Bishop method of slices to determine a factor of safety against failure. The program incorporates geogrid reinforcement by considering the geogrid tensile force as a force that produces additional rotation-resisting moment (Schmertmann, et al, 1986).

Soil strength parameters were determined by the geotechnical consultant based on previous experience with similar soils (ATEC, 1986). The following parameters were used for the wall and slope fill: ϕ' (angle of internal friction) = 28° ; c (cohesion) = 50 pounds per square foot (2 kN/m^2); γ (unit weight) = 110 pounds per cubic foot (17 kN/m^3). These parameters are typical for compacted soils in the Piedmont Physiographic Province. For the underlying soft soils, the following parameters were used: $\phi' = 25^\circ$, $C = 0 \text{ psf}$, $\gamma = 120 \text{ pcf}$ (19 kN/m^3). Ground water was assumed to be at the existing ground surface.

A surcharge equal to 70 psf (3 kN/m^2) was assumed for all cases analyzed to account for pavement and small live loads. The aim of the design was to reach a minimum acceptable factor of safety for global stability of 1.5. For the 10 foot (3.0 m) high wall, 5 layers of TENSAR® SR2 high density polyethylene uniaxial geogrids, with a minimum embedment length of 12 feet (3.7 m) were required to stabilize the soil mass. Polypropylene biaxial geogrids were used for the temporary wrap-around facing system and were placed at a vertical spacing of 1.5 feet. The biaxial geogrids used for the wall face were TENSAR® SS1 geogrids.

Due to the existing soft soil conditions, a layer of biaxial geogrid was included at the top of the existing ground to create a construction working surface. A layer of biaxial geogrid, 24 feet (7.3 m) long was placed 1 foot (0.3 m) below the bottom of the wall to help minimize differential settlement.

For global stability, a layer of uniaxial geogrid was placed 2 to 3 feet (0.6 to 0.9 m) below the bottom of the wall. The embedment length of this layer varied from 18 to 25 feet (5.5 to 7.6 m) based on the slope height. Furthermore, it was necessary to lengthen the two bottom geogrid layers (used for the wall stability) from a minimum length of 15 feet (4.6 m) to a maximum length of 22 feet (6.7 m) depending on the slope height. Figure 2 shows a typical design cross section for the ten foot (3.0 m) high wall underlain by a 10 foot (3.0 m) high slope.

A safe working tensile stress level of 2,000 pounds per linear foot (29 kN/m) was used for the uniaxial geogrid. This value is based on long-term in-isolation creep performance. The ultimate strength of the geogrid is 5,400 (79 kN/m) pounds per linear foot. A safe working tensile stress (in the cross machine direction) of 270 pounds per linear foot (4 kN/m) was used for the biaxial geogrid. The peak tensile strength in CMD of this geogrid is 1,400 per linear foot (20 kN/m).

Drainage for the wall was provided using a continuous drainage net (TENSAR® DN1) with a light weight non-woven geotextile cover placed continuously along the wall face. This drain was tied into a perforated pipe to collect any water and outlet into weep holes in the wall face. Also, a backfill drain was placed behind the reinforced fill to keep the fill from becoming saturated. This drain was placed along the existing slope down to the wall face and tied into the wall drainage system. This drain consisted of a TENSAR® drainage composite DC1200 (a drainage net with a geotextile bonded to both sides). Details of the drainage system are also shown on Figure 2.

CONSTRUCTION

Prior to construction of the slope and wall, the existing soft subgrade was stabilized using a layer of biaxial geogrid. Before the geogrid was placed, the subgrade could not support rubber-tired construction equipment. After the geogrid was placed, a 12- to 18-inch

(30 to 46 cm) "bridge" lift of soil was placed using a track-mounted front-end loader. The soil was carefully placed ahead of the loader to keep the tracks from operating directly on the geogrid layer. Following this procedure, the area was stable enough to support rubber-tired scrapers and self-propelled sheepsfoot compaction equipment. The fill slope was constructed using on-site micaceous sandy silts or silty sands. The slope and wall fill was placed in thin lifts (6 to 8 inches loose measure) and compacted to 95% of the standard Proctor maximum dry density (ASTM D 698). Figure 3 shows compaction equipment used in wall construction. A layer of uniaxial geogrid was placed about 2 feet below the wall footing elevation (for global stability) and a layer of biaxial geogrid was placed about 1 foot below the wall footing to minimize differential settlements. When the slope was completed to the bottom elevation of the wall footing, the footing was constructed. Settlement points along the footing were established to determine the extent of settlement as the footing was constructed.

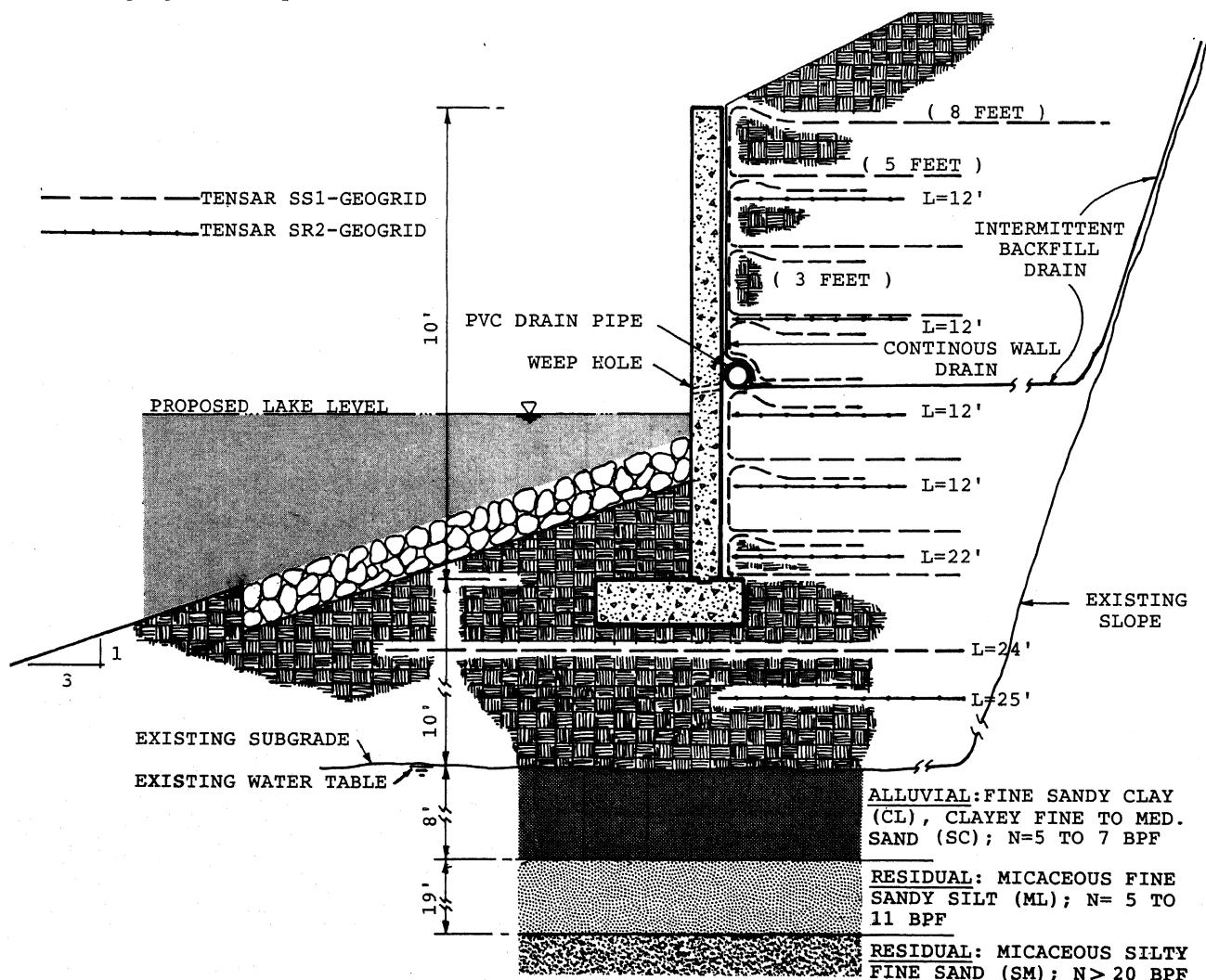


Figure 2. Typical Design Cross Section



Figure 3. Compaction of Wall Backfill

Temporary wooden forms were then erected to provide a working face for construction of the geogrid reinforced wall. The wall face was constructed by placing the bottom portion of the wrap on the subgrade and nailing the upper portion of the wrap to the forms. Figure 4 shows the wooded forms with the geogrid and geotextile nailed to the form. After placement and compaction of the required fill depth, the upper portion of the wrap was pulled down and tensioned using pitch forks. Figure 5 shows the upper wrap being tensioned and soil being placed as the wall was continued. At the specified elevations, the main wall reinforcement, TENSAR® SR2 geogrid was placed perpendicular to the wall face. Figure 6 shows the main reinforcement being placed.

The backfill drain and perforated pipe were placed at the specified elevation. The wall drain for the large radius portion of the wall was included inside the facing wraps. For the small radius portion, the wall drain was placed against the geogrid face prior to constructing the concrete face.

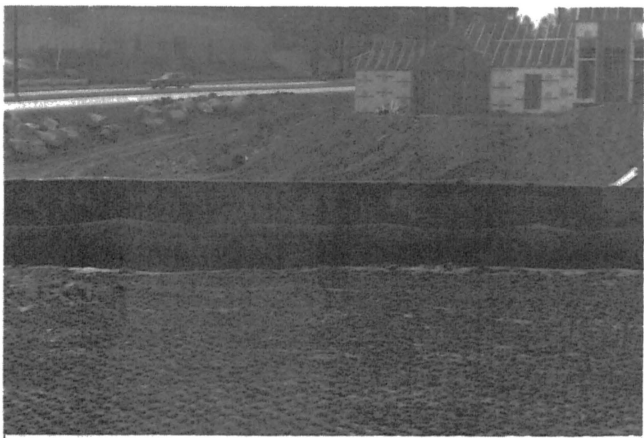


Figure 4. Facing Wrap Attached to Temporary Forms



Figure 5. Tensioning of Upper Facing Wrap

PERFORMANCE

When the reinforced wall was completed, settlement measurements were made on the wall footing for approximately one month. Approximately 2 inches of settlement was recorded during the monitoring period. At that time, the settlement was essentially complete and the forms were removed. The geogrid reinforced wall provided a nearly vertical face while the concrete facing was constructed. Figure 7 shows the geogrid reinforced wall with the steel reinforcement for the concrete facing in place.

The concrete facing was completed shortly after the settlement monitoring period. Figure 8 shows the completed wall.



Figure 6. Placement of Main Reinforcement

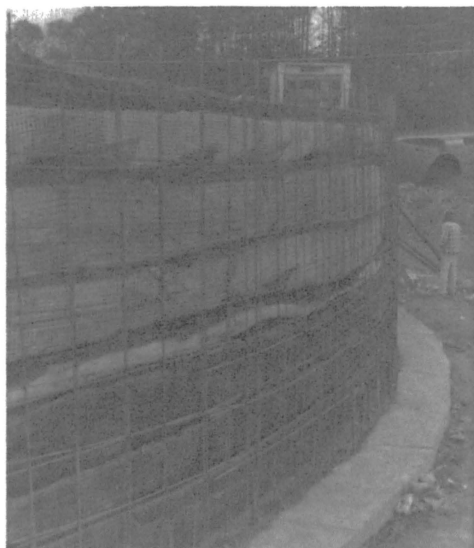


Figure 7. Completed Geogrid Reinforced Wall

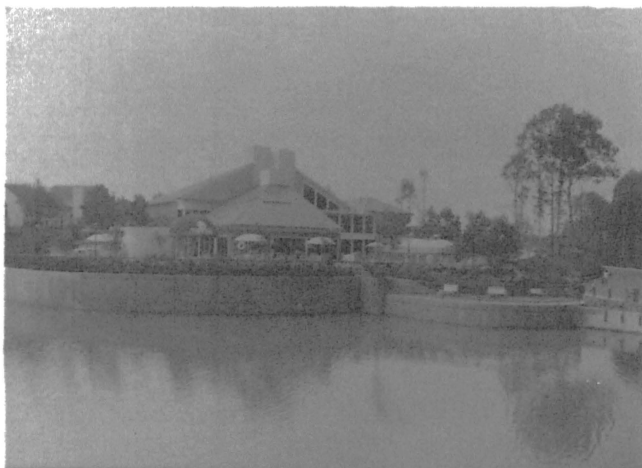


Figure 8. Completed Structure

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REFERENCES

- ATEC Associates, Inc. (1986), "Subsurface Investigation and Geotechnical Engineering Evaluation - TIMBER TRACE APARTMENTS - Stone Mountain, Georgia" ATEC Project Number 32-63108.
- Berg, R.R., Bonaparte, R., Anderson, R.P., and Chouery, V.E. (1986), "Design, Construction, and Performance of Two Geogrid Reinforced Soil Retaining Walls", Proceedings, 3rd International Conference on Geotextiles, Vienna, Austria, Vol. 2, 401-406.
- Jones, Colin J.F.P. (1985), Earth Reinforcement and Soil Structures, Butterworth and Co., Ltd.
- Schmertmann, G.R., Chouery-Curtis, V.E., Johnson, R.D. and Bonaparte, R. (1987), "Design Charts for Geogrid-Reinforced Soil Slopes", Geo-Synthetics '87 Conference Proceedings, New Orleans, Louisiana, Vol. 1, 108-120.
- The TENSAR Corporation (1986), "Guidelines for the Design of TENSAR Geogrid Reinforced Soil Retaining Walls", TENSAR Technical Note RW1.

CONCLUSIONS

Because of underlying compressible soils, a conventional concrete cantilever retaining wall was not feasible for this project. Alternatives included a deep foundation system for the cantilever wall or a flexible wall that would tolerate the expected settlements. A geogrid reinforced wall with an underlying reinforced slope was chosen as the most cost-effective solution. Geogrids were used to stabilize the existing soft subgrade, reinforce the slope and wall fill, and provide flexible facing elements for the wall. After settlement monitoring of the geogrid reinforced wall, a concrete facing was constructed. The geogrid reinforced wall performed as expected; tolerating the settlement and providing a temporary vertical face.